**IX.		EARTHQUAKE SAMPLING (Revised 8/21/01)**

**A.	Introduction**
The bridge unit has pretty much drawn a line right down Route 141 with everything to the east of 141 and including 141 requiring earthquake samples.   We need 1 to 2 holes per structure.

**B.	Sampling Procedure**
3" Sample	2.5'
Split Spoon	1.5'
Clean Out to 5' Soil	
3" Sample	2.5'
Split Spoon	1.5'
Clean Out to 10'	
3" Sample	2.5'
Split Spoon	1.5'
Clean Out to 15'	

Use the above sampling procedure to a minimum of 50 feet. If the soil becomes too hard for undisturbed sampling or sand is encountered, continue penetrating every 5' to a minimum depth of 50 feet. The depth should be increased if loose sands or soft cohesive soils are encountered. Samples should be taken from the penetrations. Water tables are important. This sample hole may also count as a bridge boring since you are penetrating.

**C.	Samples Needed**
1.	Qu- needed for undrained shear strength and allowable bearing -one per layer.
	Soil (typically 3" diameter and 8" minimum length).
	Rock (typically 2" diameter and 5" minimum length).
2.	Moistures-one per layer, may be taken from split spoon (minimum 100g).
3.	Atterberg limits-one per layer, may be taken from split spoon (minimum of about 500g).
4.	Direct Shears-needed for wall design- one per layer (typically 3" to 5" diameter and 6" minimum length).
5.	Gradations-one per layer for silts, sandy, and gravelly soils. Additional samples are necessary if the stiffness in the case of silts or density in the case of sandy soils changes. Sieves commonly used are 3/4",3/8",No.4,No.10,No.16, No. 40, No. 50, No. 100, and No. 200. (Normally a 1000 grams is required, but since the normal procedure is to obtain samples with a split spoon, 400g is acceptable.)

D. Reporting

1. You may use regular bridge logs if you are not concerned with liquefaction.

    a. Report Poisson's ratio in cover letter as published in FHWA/RD-86/102 Seismic Design of Highway Bridge Foundations.

        i.   0.45 for CL, CH, and ML
        ii. 0.35 for sand

    b. Report % Passing the # 200 sieve for English units or 75um (micro meters) for metric.

        Ex.   Soil Classification Test Data
        Depth, m   LL PI   ASTM Class   % Passing # 75um

    c. Report horizontal acceleration due to gravity as published in the most recent AASHTO Standard Specifications for Highway Bridges. (St. Louis is 0.1g).

2. If liquefaction is a concern, use earthquake summary sheet form (see Appendix E) and the program LIQUFAC (1991) Liquefaction is a concern when you have cohesionless soils such as sands and some silts.

    a. Dr= Relative Density (for sand or gravel only)
        DM 7.1 –87 or FHWA/RD-86/102, page 19

    b. Undrained Shearing Strength (U.S.S.) kPa or pcf (for cohesive soils only)
        U.S.S. = Qu/2=Torvane (for split-spoon or very stiff to hard cohesive samples determine Qu using pocket pentrometer and divide by 2)

Liquefaction

c. Earthquake induced Shearing Stress Ratio   (CSR)- Ri in LIQUFAC program

Shear Stress Tav $= (0.65 \times \text{Amax} \times \gamma_t \times H \times r_d)/g$ Eq. 2 FHWA/RD-86/102 page 28

Amax = max acceleration at ground surface (usually 0.1 to 0.3g)
γ_t = total unit weight of soils
H = depth to middle of soil layer
r_d = stress reduction factor For depths less than 12 m "Seed and Idriss" average values may be used. Alternatively $r_d = 1 - 0.015\ z$ (Iwasaki et al. 1978) may be used. z = depth in meters or
$r_d = 1 - 0.00457\ z$ z = depth in feet, r_d usually 0.8 to 1.0
g = acceleration due to gravity (usually taken as 1.0 g)

Usually the normalized stress ratio or Earthquake induced Shearing Stress Ratio CSR = Tav/σ_v' is used in all calculations.

σ_v' = Effective Overburden Pressure = γ' x H
H = depth to middle of layer
$\gamma'\ =\ \gamma_t - \gamma_w$

Earthquake induced Shearing Stress Ratio (CSR)

$\text{CSR} = \dfrac{\text{Shear Stress}}{\text{Effective Overburden Pressure}} = \dfrac{\text{Tav}}{\sigma_v'}$ Eq. (8-3b) FHWA-SA-97-076 p. 117

d. Resisting Stress Ratio (R. S. R.) Rf in LIQUFAC program
Found from correlating spt blow counts to charts
Note FHWA calls the Resisting Stress Ratio CSR and this is confusing. We prefer RSR

Corrected Blow Count For Sand
$(N_1)_{60} = C_n N_{60}$
C_n = constant to normalize the effects of overburden pressure
C_n From FHWA-SA-97-076 Figure 57, page 120
Or $C_n = 9.79\ (1/\sigma_v')^{1/2}$ kPa   or $C_n = 1.415\ (1/\sigma_v')^{1/2}$ ksf   (Liao & Whitman 1986)
N_{60} = Spt blow count corrected for automatic hammer energy
For $(N_1)_{60} > 30$ Liquefaction Not applicable

From Figure 58 FHWA-SA-97-076 page 121
With magnitude of Earthquake given (7.5)
Resisting Stress Ratio = $\text{CSR}_{m=7.5}$ or RSR from Figure 58

e. Factor of Safety for Liquefaction (F. S. Liqu)
Fs=RSR/CSR = Rf/Ri
CSR = Ri = Earthquake induced shearing stress ratio
RSR = Rf = Resisting Stress Ratio

f.   Gmax (kpa or tsf) = Maximum Shear Modulus
Low strain dynamic shear modulus measured at shear strain
amplitude of less than 0.001 % however shearing strains due to
earthquakes range from 0.01 to 0.5 % and may reduce Gmax by
0.9 to 0.2 Gmax

Gmax can also be attained from correlations of overburden
stress or from standard penetration tests.

$Gmax = 240N_{60}{}^{0.8}$ (kip/ft^2.) Ohsaki & Iwasaki 1973
$Gmax = 12,000N_{60}{}^{0.8}$ (kPa)

Reliability of such coorelations are low and it is preferable to
determine Gmax in the field. The Seismic Cone Penetration Test
is used to determine the Shear velocity(Vs). Gmax may then be
determined by the following equation $Gmax = \rho Vs^2$

g.   Vs (m/s or ft/s) = Shear Velocity
S or shear waves cause shearing deformation in a material during
seismic events. Shear waves can be measured directly with a
seismic cone penetration test or by crosshole tests. Shear wave
velocity can be calculated from the shear modulus G.
      Vs=Square Root (Gmax/ρ),
      ρ=soil density or from correlations to standard
      penetration tests. $\rho = \gamma_t / g$

The LIQUFAC program uses the coorelation of shear wave
velocity with SPT and effective verticle stress (Figure 4) to
determine the shear wave velocity.

h .   G (kPa or tsf)= Shear Modulus reduced due to seismic event.
Shear modulus is used in calculating stiffness values for footings.
As the shear strain of the soil increases during seismic events, the
shear modulus decreases. LIQUFAC uses the equation

$\gamma_{cyc} = 0.65$ x Amax x σ'o x r$_d$ / [g x Gmax x (G/Gmax)]
γ_{cyc} = cyclic shear strain
Remaining terms previously defined

And the coorelation of G/Gmax vs. Cyclic Shear Strain (Figure
8) to calculate the cyclic shear strain and G/Gmax ratio using
iterations. Once the G/Gmax ratio is know the Shear Modulus or
G may be calculated

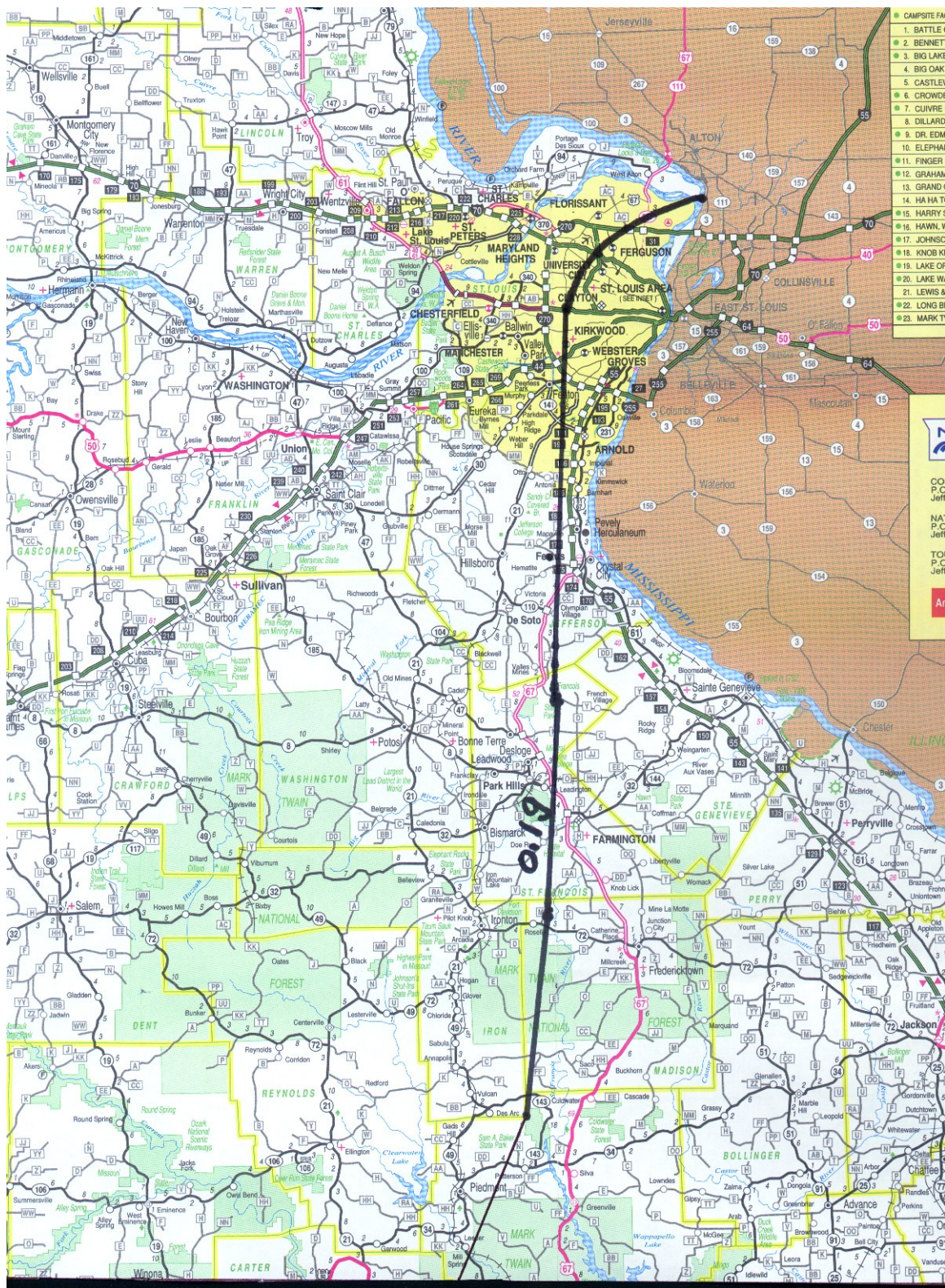The shear modulus may also be calculated from seismic response analysis programs such as SHAKE91.

i. Es (kPa or tsf) = Youngs Modulus of Elasticity Es can be obtained from cone penetration tests and/or flat plate dilatometer tests, or calculated if the shear modulus and Poisson's ratio are known.
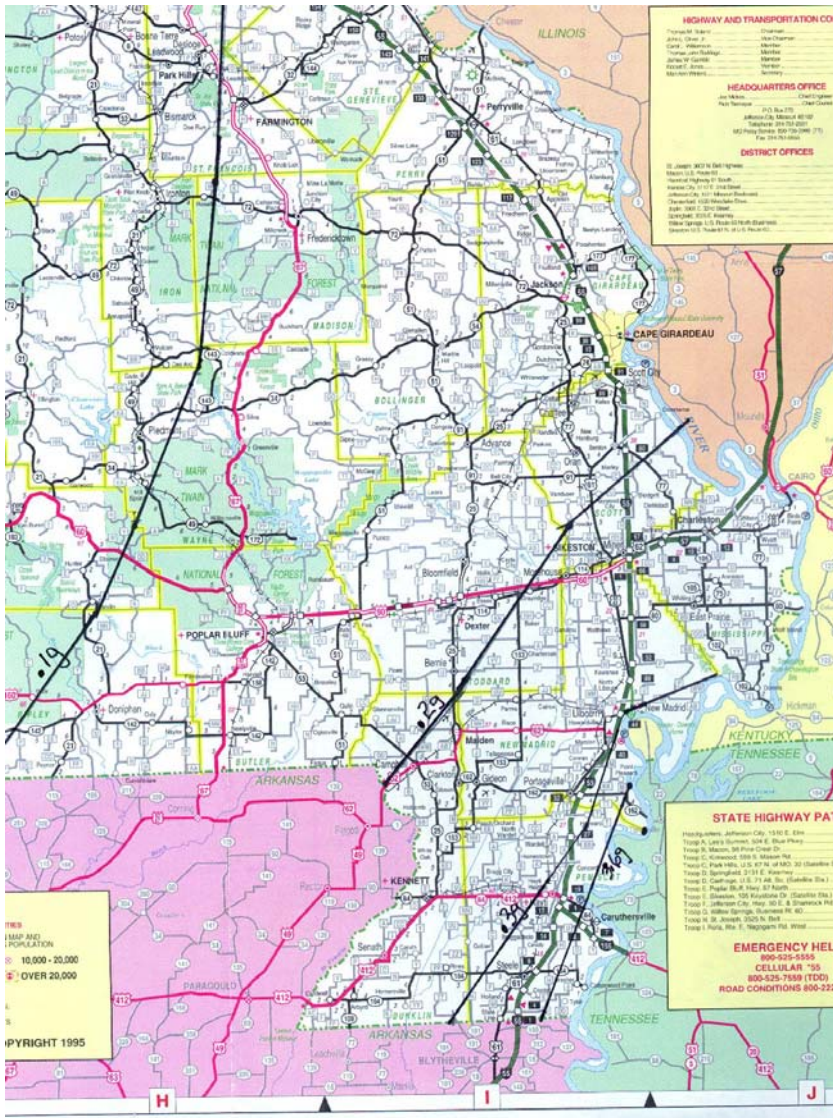
Es = 2(1+υ)G

G = shear modulus

v= Poisson's ratio

v = 0.45 for cohesive soils
  = 0.35 for cohesionless soils

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Steele

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ARKANSAS
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TENNESSEE

MAP AND
POPULATION
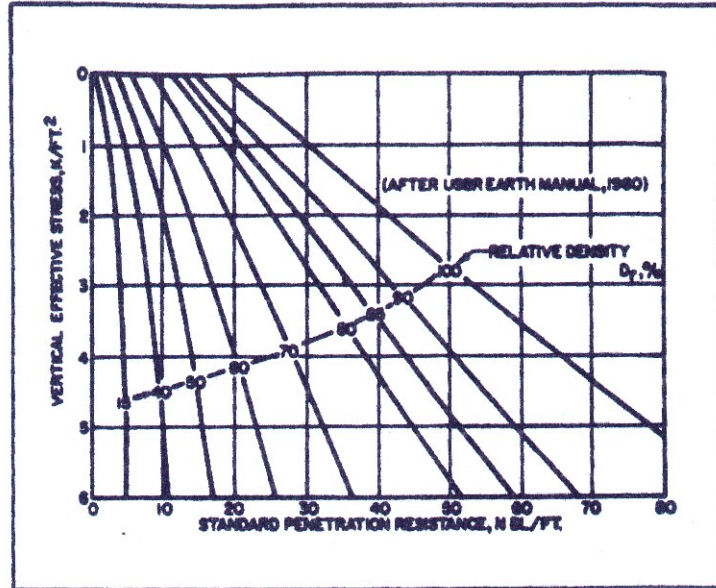10,000 - 20,000
OVER 20,000

H   I   J

FIGURE 3
Correlations Between Relative Density and Standard Penetration
Resistance in Accordance with Gibbs and Holtz

DM 7.1-87

Relative Density and Angle of Internal Friction (ϕ) for Cohesionless Soils[11,12]

Type of Soil	Resistance N (blows/ft)	Relative Density D_r	Angle of Internal Friction ϕ (Deg)	
			Peck et al.[11]	Meyerhof[12]
Very loose sand	<4	<0.2	<29	<30
Loose sand	4-10	0.2-0.4	29-30	30-35
Medium sand	10-30	0.4-0.6	30-36	35-40
Dense sand	30-50	0.6-0.8	36-41	40-45
Very dense sand	>50	>0.8	>41	>45

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Undrained Shearing Strength of Cohesive Soils[13]

Penetration Resistance N (blows/ft)	Undrained Shear Strength c (kips/ft^2)*	Consistency
<2	<0.25	Very soft
2-4	0.25-0.50	Soft
4-8	0.50-1.00	Medium
8-15	1.00-2.00	Stiff
15-30	2.00-4.00	Very stiff
>30	>4.00	Hard

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If the results of a seismic site response analysis are available, CSR_{EQ} can be evaluated from τ_{max} as:

$$CSR_{EQ} = 0.65\, \tau_{max}/\sigma_v{}'$$

(8-3b)

Note that the ratio $\tau_{max}/\sigma_v{}'$ corresponds to the peak average acceleration denoted by k_{max} in chapter 6.
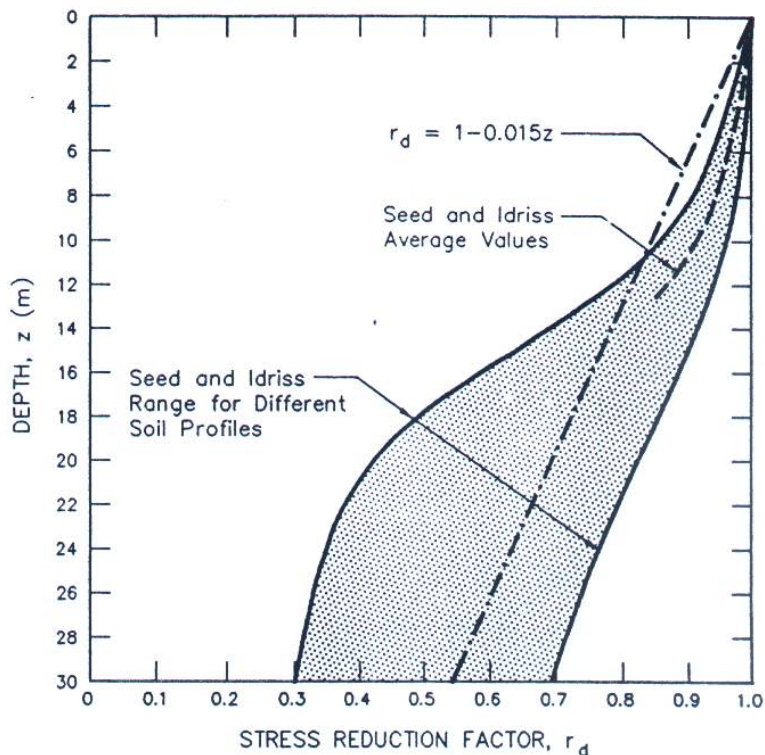

Figure 56. Stress reduction factor, r_d (modified after Seed and Idriss, 1982, reprinted by permission of EERI).
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Step 5:   Evaluate the *standardized* SPT blow count, N_{60} which is the standard penetration test blow count for a hammer with an efficiency of 60 percent (60 percent of the nominal SPT energy is delivered to the drill rod). The "standardized" equipment corresponding to an efficiency of 60 percent is specified in table 8. If nonstandard equipment is used, N_{60} is obtained from the equation:

$$N_{60} = N \cdot C_{60}$$

(8-4)

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Stress Reduction Factor
(From Seed and Idriss, 1982)

Depth, ft

Average Value

Range for Different Soil Profiles

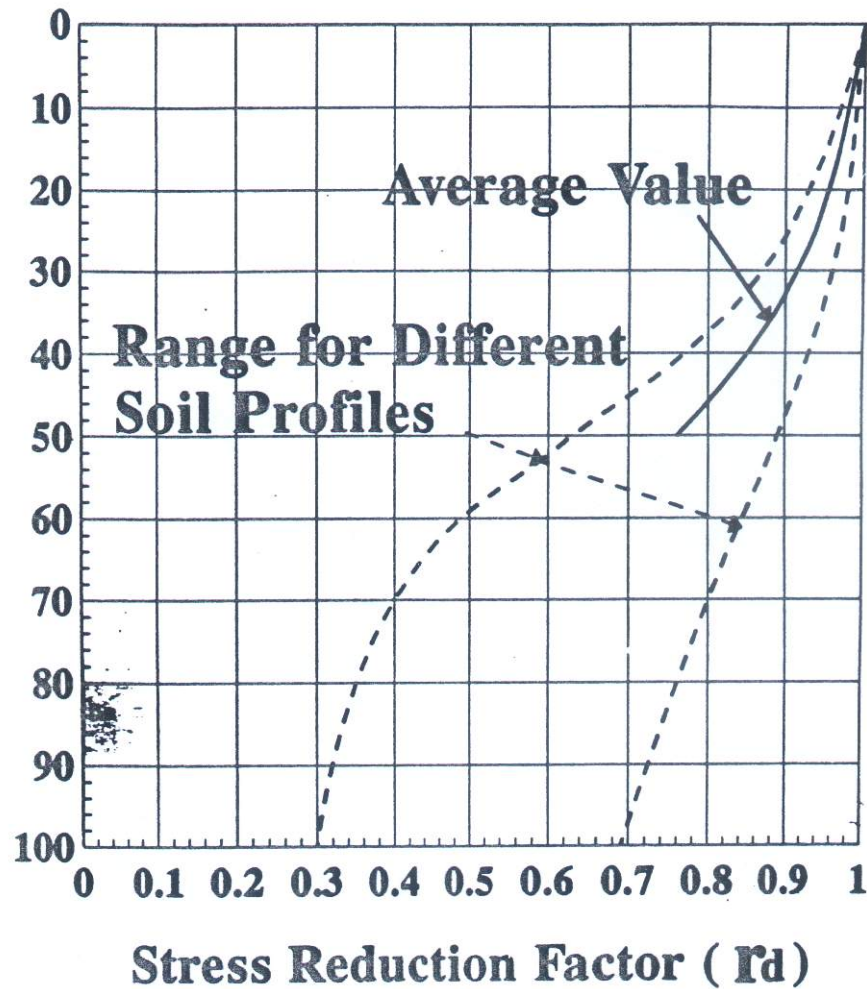**Stress Reduction Factor (r_d)**

Fig. 5 Range of stress reduction factor, r_d, for different soil profiles

Step 6:   Calculate the normalized standardized SPT blow count, $(N_1)_{60}$. $(N_1)_{60}$ is the standardized blow count normalized to an effective overburden pressure of 96 kPa in order to eliminate the influence of confining pressure. The most commonly used technique for normalizing blow counts is via the correction factor, C_N, shown in figure 57 (Seed et al., 1983). However, the closed-form expression proposed by Liao and Whitman (1986) may also be used:

$$C_N = 9.79 \ (1/\sigma_v')^{1/2} \tag{8-5}$$

where σ_v' equals the vertical effective stress at the sampling point in kPa.
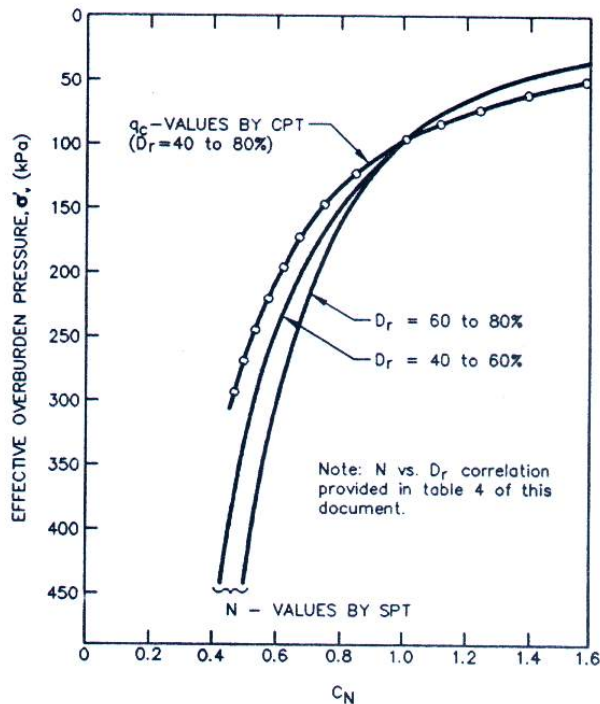
Figure 57. Correction factor for the effective overburden pressure, C_N (Seed et al., 1983, reprinted by permission of ASCE).

As shown in figure 57, the Seed et al. (1983) effective overburden correction factor curves are valid only for depths greater than approximately 3 m (approximately 50 kPa). A similar plot presented by Liao and Whitman (1986) suggests that C_N in equation 8-5 should be limited to 2.0 at depths lower than 3 m.

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Regardless of the manner in which C_N is estimated, the normalized standardized blow count is calculated as:

$$(N_1)_{60} = C_N \cdot N_{60} \qquad (8\text{-}6)$$

Other factors, such as grain size distribution, may influence C_N (Marcuson and Bieganousky, 1977). However, considering the uncertainties involved in the SPT itself, the application of equipment and overburden pressure correction factors should be sufficient for engineering purposes.

Step 7: Evaluate the critical stress ratio $CSR_{7.5}$ at which liquefaction is expected to occur during an earthquake of magnitude M_w 7.5 as a function of $(N_1)_{60}$. Use the chart developed by Seed et al. (1985), shown in figure 58, to find $CSR_{7.5}$.
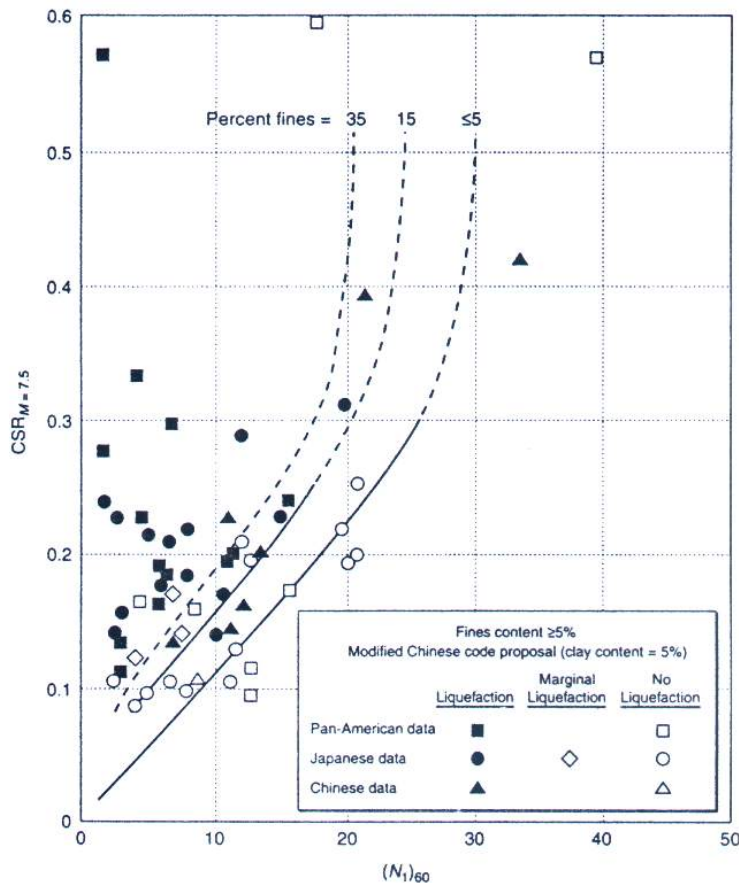

Figure 58. Relationship between stress ratio causing liquefaction and $(N_1)_{60}$ values for sands for M_w 7.5 earthquakes (Seed et al., 1985, reprinted by permission of ASCE).

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Table 6. Correlations for estimating initial shear modulus.

Reference	Correlation	Units	Limitation
Seed et al. (1984)	$G_{max} = 220 \, (K_2)_{max} \, (\sigma'_m)^{1/2}$ $(K_2)_{max} \approx 20(N_1)_{60}^{1/3}$	kPa	$(K_2)_{max} \approx 30$ for very loose sands and 75 for very dense sands; $\approx 80\text{-}180$ for dense well graded gravels; Limited to cohesionless soils
Imai and Tonouchi (1982)	$G_{max} = 15,560 \, N_{60}^{0.68}$	kPa	Limited to cohesionless soils
Hardin (1978)	$G_{max} = \dfrac{625}{(0.3 + 0.7 \, e_o^2)}(P_a \cdot \sigma'_m)^{0.5} \, OCR^k$	kPa[1]	Limited to cohesive soils P_a = atmopsheric pressure
Jamiolkowski et al. (1991)	$G_{max} = \dfrac{625}{e_o^{1.3}}(P_a \cdot \sigma'_m)^{0.5} \, OCR^k$	kPa[1]	Limited to cohesive soils P_a = atmopsheric pressure
Mayne and Rix (1993)	$G_{max} = 99.5(P_a)^{0.305}(q_c)^{0.695}/(e_o)^{1.13}$	kPa[2]	Limited to cohesive soils P_a = atmopsheric pressure

Notes: [1] P_a and σ'_m in kPa
[2] P_a and q_c in kPa

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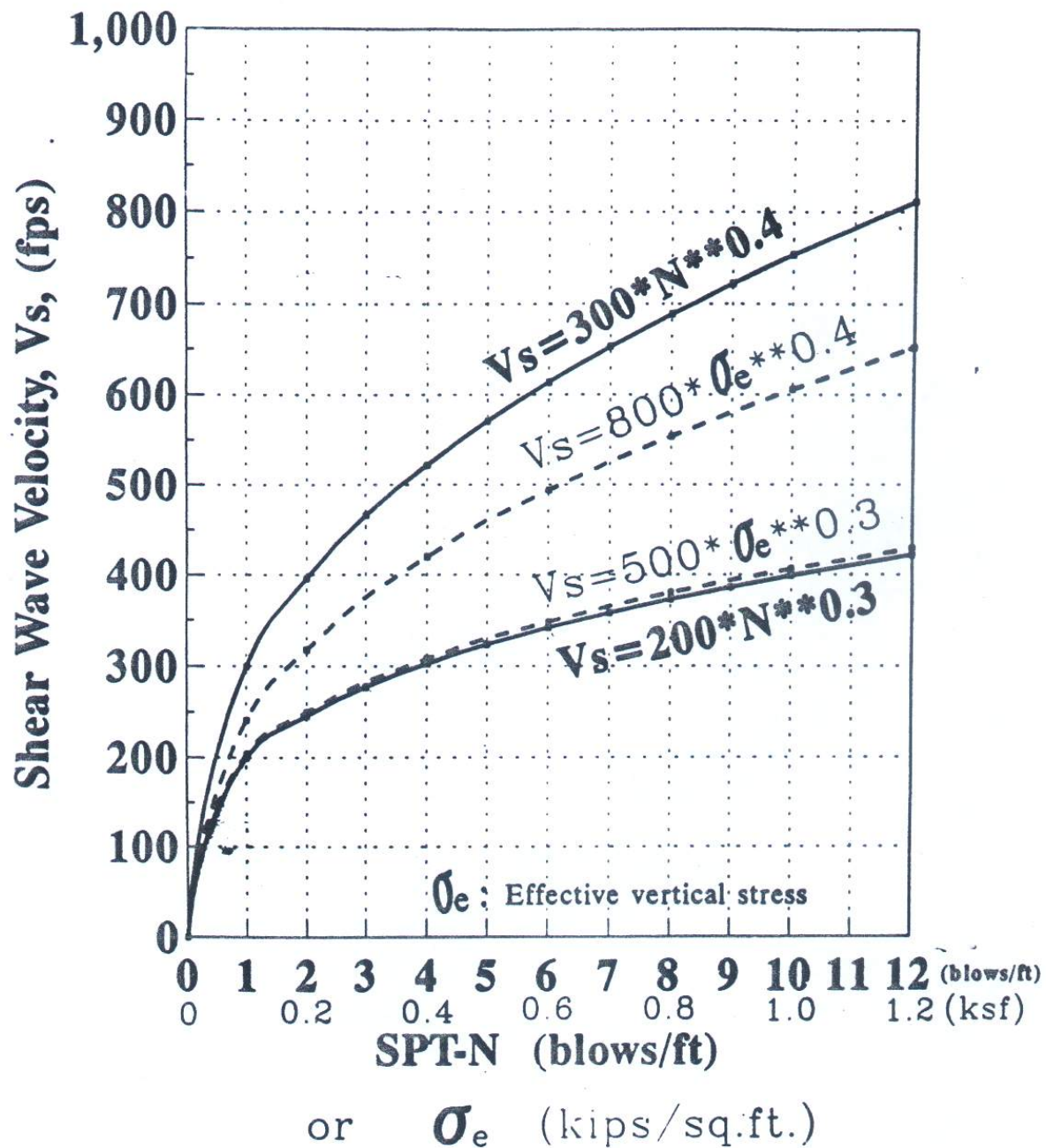
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Correlation of Vs with SPT-N & σ_e (ksf)

Fig. 4 Correlation of shear wave velocity with SPT and effective vertical stress

Relation between G/Gmax vs. Cyclic Shear Strain and Soil Plasticity of Saturated Soil
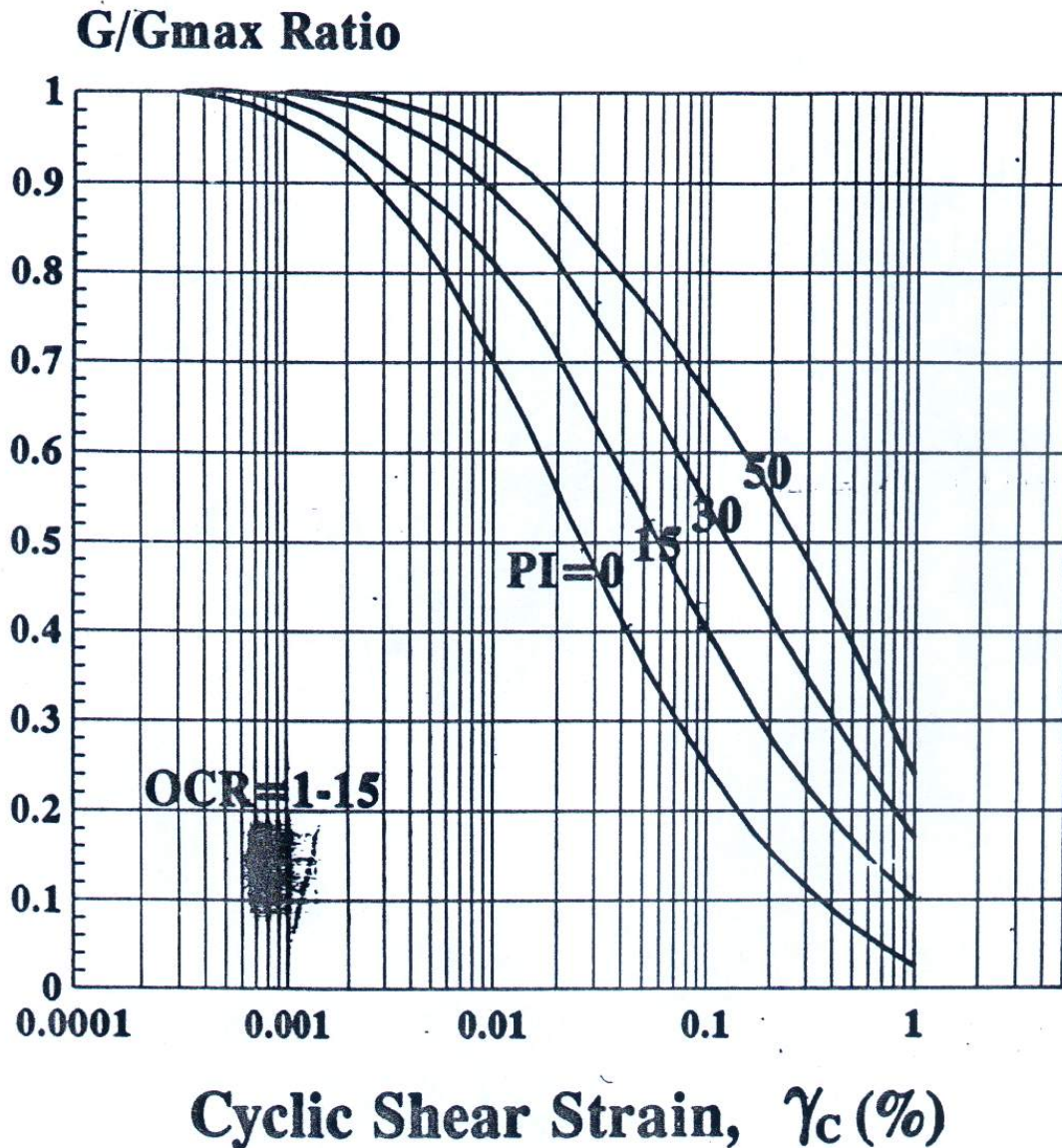## (from Vucetic and Dobry, GT Journal, Jan. 1991)

G/Gmax Ratio

labels on curves: PI=0, 15, 30, 50

OCR=1-15

Cyclic Shear Strain, γ_c (%)

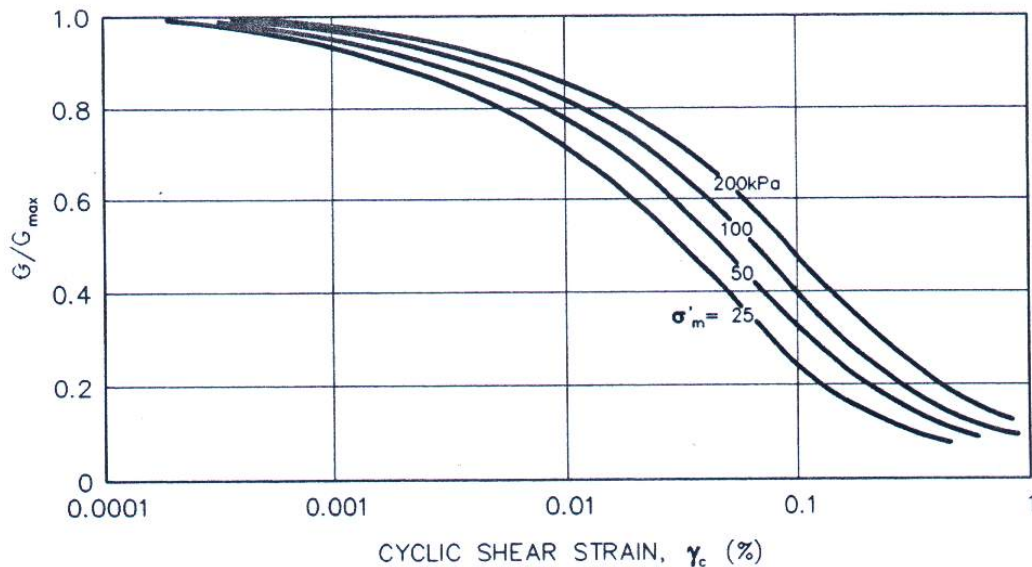Fig. 8 Influence of Plasticity Index on Shear Modulus and Cyclic Shear Stress curves of saturated soil

Figure 42. Shear modulus reduction curves for sands (Iwasaki et al., 1978, reprinted by permission of Japanese Society of Soil Mechanics and Foundation Engineering). FHWA-SA-97-076

5.4.6 Peak and Residual Shear Strength

The peak shear strength of soil not subject to strength degradation under cyclic loading may be evaluated using conventional methods, including laboratory and in situ testing and correlations with soil index properties. A key difference in seismic problems compared to static problems is that undrained strength parameters are typically used for the strength of saturated soils subjected to cyclic loading, even for cohesionless soils (e.g., sands, gravels) because of the relatively rapid rate of earthquake loading.

The dynamic undrained shear strength of a soil may be influenced by the amplitude of the cyclic deviator stress, the number of applied loading cycles, and the plasticity of the soil. For saturated cohesionless soils, even relatively modest cyclic shear stresses can lead to pore pressure rise and a significant loss of undrained strength. However, Makdisi and Seed (1978) point out that substantial permanent strains may be produced by cyclic loading of clay soils to stresses near the yield stress, while essentially elastic behavior is observed for large numbers of (> 100) cycles of loading at cyclic shear stresses of up to 80 percent of the undrained strength. Therefore, these investigators recommend the use of 80 percent of the undrained strength as the "dynamic yield strength" for soils that exhibit small increases in pore pressure during cyclic loading, such as clayey materials, and partially saturated cohesionless soils.
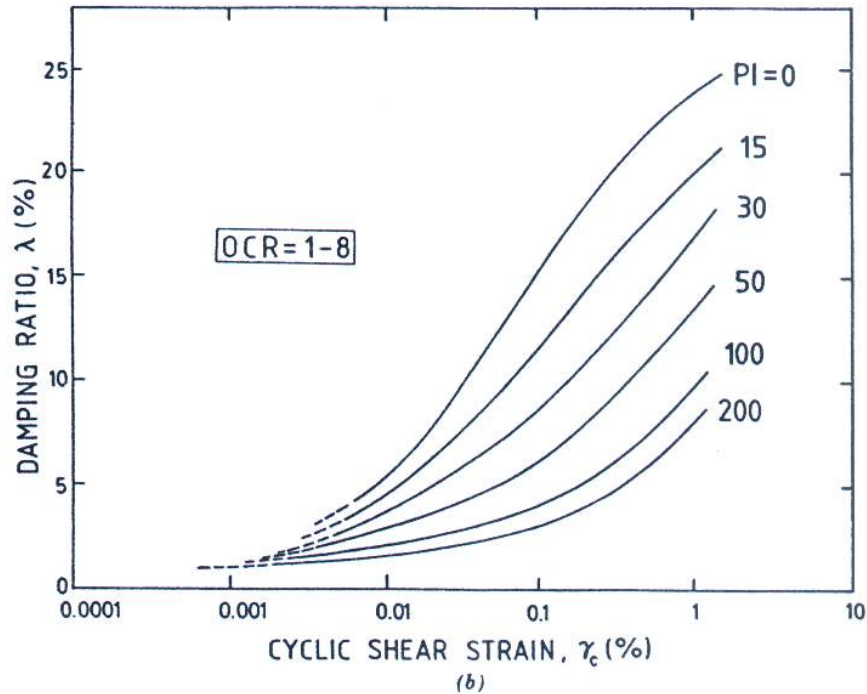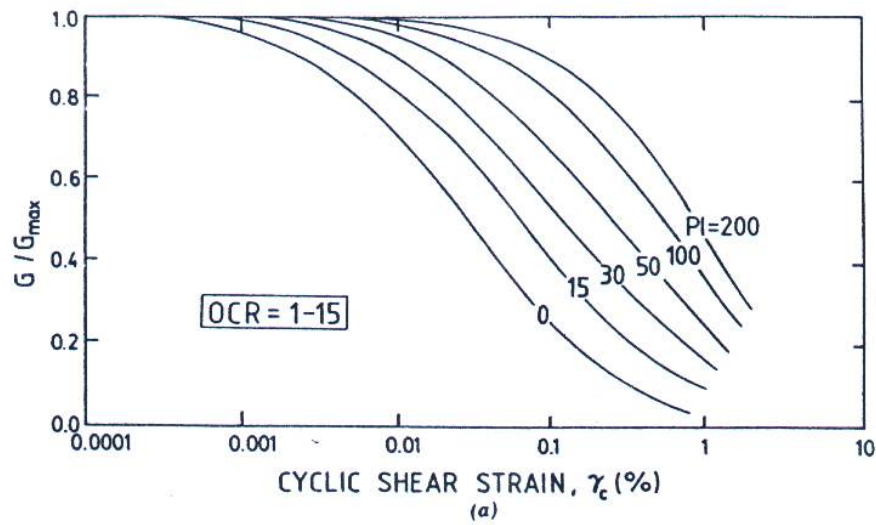
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Figure 43. Shear modulus reduction and damping ratio as a function of shear
strain and soil plasticity index (Vucetic and Dobry, 1991,
reprinted by permission of ASCE).

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